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BOSTON SOCIETY OF CIVIL ENGINEERS

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PROCEEDINGS

NOTICE OF REGULAR MEETING.

A REGULAR meeting of the Boston Society of Civil Engineers will be held on

WEDNESDAY, FEBRUARY 21, 1917,

at 8.00 o'clock P.M., in CHIPMAN HALL, TREMONT TEMPLE, BOSTON.

Mr. James W. Rollins will give a talk on "The New Boston Dry Dock." The talk will be illustrated with lantern slides.

S. E. TINKHAM, *Secretary.*

PAPERS IN THIS NUMBER.

"Flood Control in the Miami Valley," Arthur E. Morgan.

Discussion of "The Groined Arch as a Means of Concrete Floor Construction."

Reprints from this publication, which is copyrighted, may be made provided full credit is given to the author and the Society.

Contributors are hereby notified that proof will not be submitted to them for examination unless requested before the 10th of the month preceding the month of publication.

BINDING SOCIETY JOURNAL.

THE Secretary has made arrangements for binding Volume 3 of the JOURNAL of the Society. The ten numbers will be bound

in one volume, the style of binding to be uniform with that of Volumes 1 and 2. Because of the increased cost of material, the price of binding this year will be 80 cents per volume.

Numbers for binding must be sent to 715 Tremont Temple, Boston, before March 1, 1917, in order to be bound at the price named.

ANNUAL MEETING OF THE SOCIETY.

THE annual meeting of the Society will be held on Wednesday, March 21, 1917, at the Boston City Club, Ashburton Place, Boston.

The annual meeting for the transaction of business and the announcement of the result of the letter ballot for officers will be held at 12 o'clock M. A short address will be made by the retiring President.

The annual dinner will be served at two o'clock in the auditorium of the club.

In the evening, a smoker will be held in the auditorium, at which light refreshments, music, and other entertaining features may be expected.

Further details will be furnished in a special circular to be sent out early next month, and in the March JOURNAL.

MINUTES OF MEETINGS.

BOSTON, MASS., January 10, 1917. — A special meeting of the Boston Society of Civil Engineers was held this evening in Chipman Hall, Tremont Temple, and was called to order at 8 o'clock by the President, Richard A. Hale.

Mr. Jacob Lowenstein, of the engineering department of the American Bridge Company, read a most interesting paper on "The Construction and Erection of Hell Gate Bridge." The paper was profusely illustrated with lantern slides and motion pictures. At the conclusion of the reading of the paper, Mr. Lowenstein kindly answered numerous questions in relation to the erection of the bridge.

On motion of Past President Fay, the thanks of the Society were extended by a unanimous rising vote to the American Bridge Company and its representative, Mr. Lowenstein, for their courtesy in presenting to the Society so valuable and instructive a paper.

The attendance at the meeting, including both members and guests, was 375.

S. E. TINKHAM, *Secretary*.

BOSTON, MASS., January 24, 1917. — A regular meeting of the Boston Society of Civil Engineers was held this evening in Chipman Hall, Tremont Temple, and was called to order at 7.50 o'clock by the President, Richard A. Hale. In the absence of the Secretary, Mr. F. O. Whitney served as Secretary *pro tem*.

There were present 91 members and guests, including 44 ladies. (The occasion was Ladies' Night, supper being served at 6.30 P.M.)

The record of the last meeting was approved as printed in the January JOURNAL.

The President reported for the Board of Government the election of the following to membership in the Society in the grades named.

Members — Messrs. John Brazer Babcock 3d, Leslie P. Reed, George Summers Sawyer and Edward Van der Pyl.

Associate — Mr. Joseph A. Tomasello.

Junior — Mr. Freeman Hudson Horton.

The President announced the death of Walter N. Charles, who died January 20, 1917.

By vote, the President was requested to appoint a committee to prepare a memoir.

An invitation was read from the American Roadbuilders' Association inviting members of this Society to attend its convention to be held in the Mechanics Building, Boston, February 5 to 9, inclusive, and to name three delegates to represent the Society.

This invitation was accepted and the President was authorized to appoint three delegates.

The President named Messrs. Joshua Atwood, Lewis M. Hastings and William E. McClintock as delegates to the convention.

It was voted that the President appoint a committee of three to nominate a committee of five to nominate officers for the coming year.

The President appointed Messrs. Frank W. Hodgdon, Edward W. Howe and Sidney Smith a committee, who reported the names of Messrs. John E. Carty, Herbert R. Stearns, Herbert T. Gerrish, Sturgis H. Thorndike and John J. Rourke, who were elected a committee to report nominations for officers.

After welcoming the ladies, the President introduced Past President Desmond FitzGerald, who gave an interesting illustrated talk on his travels.

Adjourned at 9.30 o'clock.

FRANK O. WHITNEY, *Secretary pro tem.*

BOSTON, MASS., January 3, 1917. — A meeting of the Sanitary Section, Boston Society of Civil Engineers, was held this evening in Chipman Hall, Tremont Temple. The meeting was opened at 7.30 o'clock by the Chairman.

The first paper presented was by Raymond C. Allen, civil engineer, on the "Outfall Sewer and Pumping Station, Manchester, Mass." The paper described the engineering work required for the construction of a 14-in. cast-iron submerged outfall in Manchester Harbor, laid for a distance of over 8 000 ft. into deep water in the outer harbor. The paper also described the pumping station and storage wells with equipment.

The second paper was by Clarence A. Moore, assistant engineer, Metropolitan Water and Sewerage Board, on the "Extension of the Metropolitan Sewerage System Outfall at Deer Island, Boston Harbor." The paper included a general description of the Metropolitan sewerage districts and the construction of a temporary outfall. The principal features were the construction of the outfall extension, methods, character of the

foundation and a description of the special multiple outlets, as well as many other features of interest.

Both papers were illustrated by lantern slides. The interest shown in these subjects is indicated by the attendance, which was 64, in spite of the fact that the night was very stormy. The meeting adjourned at 9 o'clock.

FRANK A. MARSTON, *Clerk.*

APPLICATIONS FOR MEMBERSHIP.

[February 5, 1917.]

THE By-Laws provide that the Board of Government shall consider applications for membership with reference to the eligibility of each candidate for admission and shall determine the proper grade of membership to which he is entitled.

The Board must depend largely upon the members of the Society for the information which will enable it to arrive at a just conclusion. Every member is therefore urged to communicate promptly any facts in relation to the personal character or professional reputation and experience of the candidates which will assist the Board in its consideration. Communications relating to applicants are considered by the Board as strictly confidential.

The fact that applicants give the names of certain members as reference does not necessarily mean that such members endorse the candidate.

The Board of Government will not consider applications until the expiration of twenty (20) days from the date given.

CROCKER, HAROLD SIMPSON, Brockton, Mass. (Age 28, b. Brockton, Mass.) Student at Mass. Inst. of Technology, 1909-1911, and at Franklin Union evening school, 1912-1914. From 1911 to April, 1916, with city engineer of Brockton; from April, 1916, to January 1, 1917, in private practice as civil engineer in Brockton; January, 1917, was elected city engineer of Brockton. Refers to G. E. Bolling, C. R. Felton, James Hayes, Jr., and R. S. Weston.

PIKE, WALDO FRANCIS, Cambridge, Mass. (Age 23, b. Cambridge, Mass.) Graduate of Mass. Inst. of Technology, 1915, civil engineering course. During one summer vacation employed in drafting room of N. E. T. & T. Co.;

from June, 1915, to January, 1916, template maker with New England Structural Company; from January to April, 1916, inspector in beam shop of same firm; from April to June, 1916, bridge inspector, and from June to September, structural draftsman, with B. & M. R. R.; from October, 1916, to date, with Fay, Spofford & Thorndike, Boston. Refers to C. R. Berry, F. H. Fay, B. W. Guppy, R. W. Horne, J. S. Lamson and C. M. Spofford.

SHEIK, HENRY CONNOR, Boston, Mass. (Age 22, b. Boston, Mass.) Graduate of Mass. Inst. of Technology, 1915, civil engineering course; took post-graduate work at M. I. T., 1916. During summer months, from 1907 to 1913, with Coughlan & Sheik, contractors; during summers of 1914 and 1915, with Coleman Brothers, contractors; is now with National Waterproofing Company, Boston. Refers to C. B. Breed, J. W. Howard, L. J. Johnson, Dwight Porter, C. M. Spofford and G. F. Swain.

LIST OF MEMBERS.

ADDITIONS.

BREEN, ANTHONY A.....461 Somerville Ave., Somerville, Mass.
HORTON, FREEMAN H.....12 Newbern St., Jamaica Plain, Mass.
VAN DER PYL, EDWARD.....46 Summerhill Ave., Worcester, Mass.

CHANGES OF ADDRESS.

ACKERSON, HERBERT N.....167 Babcock St., Quincy, Mass.
BADGER, FRANK S.
Care J. G. White & Co., 9 Cloak Lane, Cannon St., London, E. C., England
BURNES, GEORGE R.....11 Hyde St., Revere, Mass.
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FLETT, LOUIS E.....452 Willow St., Waterbury, Conn.
HOBSON, GEORGE F.....1121 Columbia Road, N. W., Washington, D. C.
HOWARD, JOHN L.

Metropolitan Water and Sewerage Bd., 1 Ashburton Place, Boston, Mass.
KLEINERT, ALBERT E., JR.....38 Hamilton Road, West Somerville, Mass.
MANLEY, LAURENCE B.....754 Bourse Bldg., Philadelphia, Pa.
PEACOCK, FRANK E.....200 Bay State Road, Boston, Mass.
SHERMAN, EDWARD C.

Bureau of Yards and Docks, Navy Dept., Washington, D. C.

DEATH.

CHARLES, WALTER N.....January 20, 1917

EMPLOYMENT BUREAU.

THE Board of Government maintains an employment bureau for the Society, to be a medium for securing positions for its members and applicants for membership, and also for furnishing employees to members and others desiring men capable of filling responsible positions.

At the Society rooms two lists are kept on file, one of *positions available* and the other of *men available*, giving in each case detailed information in relation thereto.

MEN AVAILABLE.

No. 388. Age 31. Student for two and one-half years at University of Maine, civil engineering course. Has had about seven years' experience, chiefly on surveys, including railway location; has served as rodman, instrumentman, chief-of-party, draftsman, and resident engineer on railroad construction; has had some experience in municipal work. Desires position on construction work with private engineer in or near Boston. Salary desired, \$25 per week.

No. 389. Age 23. High school education. Has had five and one-half years' experience in surveying, making all field and office calculations and maps; worked up from position of chainman to that of chief-of-party; has worked one and one-half years as engineer on building construction. Desires position as transitman and time-keeper. Salary desired, \$17 to \$18 per week.

No. 390. Age 33. Graduate of Tufts College, 1906, civil engineering course, degree of B.S. Experience includes three and one-half years in engineering department, B. & M. R. R.; two and one-half years' cemetery work; three years in city engineering department, two and one-half of the three as city engineer; one and one-half years as superintendent of streets. Desires work on construction, preferably road work. Minimum salary desired, \$1 800 per year.

No. 391. Age 23. Graduate of Rhode Island State College, 1916, civil engineering course, degree of B.S. Has had seven months' experience with maintenance of way department of railroad, chiefly as rodman and as transitman in charge of field parties; has also had some office experience in tracing, drafting, etc. Desires position as transitman. Minimum salary desired, \$1 000 per year.

No. 392. Age 45. Private and high school education. Experience includes six years on street work in city surveyor's office; six months with gas light company on location of pipes, valves, etc.; three years with civil engineer on surveys; and about twelve years in private practice, making surveys for new buildings.

No. 393. Age 34. Graduate of University of Maine, degree of B.S.,

1906; degree of C.E., 1913. Experience includes six months of municipal and six months of railroad engineering in New Hampshire; six years with U. S. Reclamation Service in Idaho on drafting, surveys and construction; two years on general building construction and foundations in Boston and Cambridge; and two years as draftsman and superintendent on water supply and sewage disposal in Maine and Massachusetts. Desires position as engineer on operation or construction of public utility. Salary desired, \$110 per month.

LIBRARY NOTES.

RECENT ADDITIONS TO THE LIBRARY.

U. S. Government Reports.

Analysis of Silicate and Carbonate Rocks. W. F. Hillebrand.

Annual Report of Chief of Weather Bureau for 1915-16.

Annual Report of Interstate Commerce Commission for 1916.

Annual Report of Superintendent, Coast and Geodetic Survey, for 1915-16.

Artesian Water for Irrigation in Little Bitterroot Valley, Montana. Oscar E. Meinzer.

Estimates of Population of United States, 1910 to 1916, inclusive.

Geographic Tables and Formulas. Samuel S. Gannett.

Geology of Upper Stillwater Basin, Stillwater and Carbon Counties, Montana. W. R. Calvert.

Gold, Silver, Copper and Lead in South Dakota and Wyoming in 1915. Charles W. Henderson.

Gold, Silver, Copper, Lead and Zinc in New Mexico and Texas in 1915. Charles W. Henderson.

Gold, Silver, Copper, Lead and Zinc in Utah in 1915. V. C. Heikes.

Gypsum in Southern Part of Bighorn Mountains, Wyoming. Charles T. Lupton and D. Dale Condit.

Measurement of Silt-Laden Streams. Raymond C. Pierce.

Public Road Mileage and Revenues in New England States, 1914.

Public Road Mileage and Revenues in United States, 1914.
Sugar Pine. Louis T. Larsen.

State Reports.

Illinois. Bridge Manual for County Superintendents of Highways, Resident Engineers and Inspectors. Clifford Older.

Illinois. Report of State Board of Health on Sanitary Investigations of Illinois River and Its Tributaries, 1901. Gift of W. S. Johnson.

New Jersey. Revision of Primary Levels and List of Bench Marks in Northern New Jersey. C. C. Vermeule.

Ohio. Miami Conservancy District: Official Plan for Protection of District from Flood Damage, Vol. II; Report of Chief Engineer, 1916, 3 vols. bound in one. Gift of Arthur E. Morgan.

Oregon. Rules, Regulations, Forms and Practice of Office of State Engineer and State Water Board Relative to Control, Distribution and Use of Water Resources of Oregon. John H. Lewis and Percy A. Cupper.

Municipal Reports.

Atlanta, Ga. Annual Report of Water Commissioners for 1915.

Atlantic City, N. J. Annual Report of Water Department for 1915.

Denver, Colo. Annual Report of Department of Improvements for 1915.

Detroit, Mich. Annual Report of Department of Parks and Boulevards for 1916.

Kansas City, Mo. Annual Report of Fire and Water Commissioners for 1915-16.

Los Angeles, Cal. Complete Report on Construction of Los Angeles Aqueduct, 1916.

New York, N. Y. Report of Department of Water Supply, Gas and Electricity on Water Supply System, 1916.

Pasadena, Cal. How Other Cities in United States are Disposing of Their Sewage. T. D. Allin and R. V. Orbison.

Miscellaneous.

American Public Health Association: Reports and Papers, 1905-07. 6 vols. Gift of W. S. Johnson.

Canada, Department of Mines: Report on Production of Spelter in Canada, 1916. Alfred W. G. Wilson.

Handbuch für Specielle Eisenbahn-Technik. Edmund Heusinger von Waldegg. 4 vols. Gift of E. C. Sherman.

Portland Cement Association: Concrete Facts about Concrete Roads; Concrete Sewers; Concreting in Cold Weather; Farmer's Handbook on Concrete Construction.

Spray Engineering Co.: Washing and Cooling Air for Steam Turbine Generators.

Viaduct Works' Handbook. Henry N. Maynard. Gift of E. C. Sherman.

Duplicates to which members may help themselves will be found, as usual, on the ends of the tables in the reading-room. The list is as follows:

American Society of Civil Engineers: Final Report of Special Committee on Rail Sections, 1910; Index to Transactions, Vols. 22-27, 1893, and 46-59, 1901-07; Proceedings, August, 1911; September-December, 1913; January, February, August-November, 1914; January, February, April, 1915.

Illinois State Board of Health: Report of Sanitary Investigations of Illinois River and Its Tributaries, 1901.

Iowa State College, Engineering Experiment Station: Bulletin 43, Practical Handling of Iowa Clays.

Ohio, Miami Conservancy District: Official Plan for Protection of District from Flood Damage, Vols. 1 and 3. (Several copies of each.)

Ohio State Board of Health: Preliminary Report of Investigation of Rivers and Deep Ground Waters of Ohio as Sources of Public Water Supply, 1897-1898.

Massachusetts: Reports of Joint Board on Improvement of Charles River, April, 1894, and May, 1896. (Two copies of each.)

Massachusetts, Joint Board on Metropolitan Improvements: Final Report, 1911.

Massachusetts, Commission on Metropolitan Improvements: Report on Public Improvements for Metropolitan District, 1909.

Massachusetts, Harbor and Land Commissioners: Annual Reports for 1898, 1899, 1901, 1903-10, 1912.

Massachusetts, Metropolitan Park Commission: Annual Reports for 1896, 1897, 1900-1907, 1909-12.

Massachusetts, Metropolitan Sewerage Commission: Annual Reports for 1895-96; Report upon High Level Gravity Sewer for Relief of Charles and Neponset River Valleys, 1899.

Massachusetts, State Board of Health: Report on Sewerage of Mystic and Charles River Valleys, January, 1889.

United States, Bureau of Mines: Accidents at Metallurgical Works in United States during Calendar Years 1913 and 1914, compiled by A. H. Fay; Analyses of Coals, Part II, by N. W. Lord and others; Manufacture and Uses of Alloy Steels, by Henry D. Hibbard; Methods for Determining the Water in Petroleum and Its Products, by Irving C. Allen and Walter A. Jacobs; Prevention of Accidents from Explosives in Metal Mining, by Edwin Higgins; Saving Fuel in Heating a House, by L. P. Breckenridge and S. B. Flagg.

LIBRARY COMMITTEE.

NEW ENGINEERING WORK.

(Under this head a brief description of new engineering work contemplated or under construction will be presented each month. Engineers and contractors are requested to send descriptions of their work to the Secretary, 715 Tremont Temple, Boston, before the 1st of each month.)

Commonwealth of Massachusetts.—METROPOLITAN WATER AND SEWERAGE BOARD. — *Sewerage Works.* — Work in progress: Section 98, Dedham, Wellesley extension; Section 102, Wellesley extension (suspended for winter).

Work contemplated: Sections 99, 100, 101, Wellesley extension; Sections 76-75, Reading extension (6 000 ft. rock tunnel).

METROPOLITAN PARK COMMISSION. — *Charles River Reservation*. — The work of building reinforced concrete bridge over the Charles River at North Beacon St., Boston and Watertown, is in progress. A. G. Tomasello, contractor.

Mystic Valley Parkway. — The work of grading extension of Mystic Valley Parkway from Mystic St. to Medford St., Arlington, is in progress. James H. Fannon, contractor.

BOSTON TRANSIT COMMISSION. — *Dorchester Tunnel* has been practically completed from its beginning at Tremont St. to a point under Dorchester Ave. near Broadway in South Boston.

Work is progressing on the Broadway Station, which extends in Dorchester Ave. from Broadway to West Fourth St.

Work on Section G, which extends in Dorchester Ave. from West Fourth St. to Old Colony Ave., is substantially completed north of the N. Y., N. H. & H. R. R. Co. cut, except the resurfacing of the street. Work at and south of the railroad cut is now in progress.

Section H and Section H Extension, which are completed, extend in Dorchester Ave. from Old Colony Ave. to near Dexter St.

The next section, Section J, extending from a point near Dexter St. in a southerly direction in and near Dorchester Ave. and Boston St. to near Ralston St., and embracing the station in Andrew Square, is being constructed. The work is about three fourths done, the work remaining being in Boston St. and in the station south of Dexter St.

COMMISSION ON WATERWAYS AND PUBLIC LANDS. — *Mystic River Dredging*. — The shoal place in the Mystic River between Chelsea and Malden bridges is being dredged to the depth of 30 ft. under a contract with the Maryland Dredging and Contracting Co., which calls for a total of 1 750 000 cu. yds. About 500 000 cu. yds. have been dredged, and the work is proceeding at the rate of about 125 000 cu. yds. per month. This work is being done with scoop dredges, and the material is being taken to East Boston and used for filling purposes there.

East Boston. — Material from the shoals in Mystic River

and also material from private dredging in various parts of the harbor is being brought to East Boston and dumped in two areas, whence it is rehandled by an hydraulic dredge to an area between the new bulkhead and Wood Island. An average of about 225 000 cu. yds. per month is being deposited in this way.

Dry-dock at South Boston. — The cofferdam has been completed and the dry-dock area has been pumped out with the exception of a small pool at the inner end. A steam shovel is at work excavating in the outer portion. A track has been constructed leading down into the pit. The pumping equipment has been installed and is in operation for keeping the excavation free from water.

State Pier at New Bedford. — The State Pier at New Bedford is completed except paving. Steel shed on pier erected, second floor of concrete is laid, first floor to be laid as weather permits.

Waterway between Taunton River and Massachusetts Bay. — Two parties in field-making survey for a proposed waterway between Taunton River and Massachusetts Bay.

Conservation of Water Sources. — One party in field-taking levels in connection with the investigation of the conservation of the water sources of the state.

City of Boston. — PUBLIC WORKS DEPARTMENT. — HIGHWAY DIVISION, PAVING SERVICE.

Hill Top St.,	Granite Ave. to Hallet St.	Bituminous macadam.
Ralston St.,	Dorchester Ave. to Old Colony Ave.	Filling.

New York, New Haven & Hartford R. R. Co. — *Boston, Mass., Southampton Street Engine Facilities.* — Work consists of new thirty-nine stall engine house, 95-ft. turntable, ash pits, boiler house, office building, water tank and necessary trackage.

During past month have been excavating for engine house, office building and turntable foundations; driving concrete piles for engine house and wooden piles for turntable; pouring concrete for foundation of engine house.

Fore River Shipbuilding Corporation, Quincy, Mass., has the following work in progress:

Twenty-six submarines for United States Government.
Eight torpedo boat destroyers for United States Government.

One molasses boat for Cuba Distilling Company.

Two oil tankers for the Texas Company.

Two oil tankers for the Mexican Petroleum Company.

Four freight boats for Edgar F. Luckenbach.

Two freight boats for Fore River Shipbuilding Corporation.

One oil tanker for Argentine Government.

BOSTON SOCIETY OF CIVIL ENGINEERS
FOUNDED 1848

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications

FLOOD CONTROL IN THE MIAMI VALLEY.

BY ARTHUR E. MORGAN.*

THE map of Ohio shows the Miami River in the southwest part of the state, emptying into the Ohio River near Cincinnati, and as far upstream as Dayton, and also the Mad River beyond, extending in a direct line from southwest to the northeast. Practically all important tributaries enter from the west of this line, the smaller ones having very steep slopes, with the result that during floods the water rushes down their valleys and collects in the flatter valleys of the larger streams. The crest of the 1913 flood was caused, not so much by the movement of the water through the main valleys, as by the sudden emptying of these smaller tributaries into the larger streams. It arrived at about the same time at widely separated points on the river, and in some cases where the emptying of the tributaries was particularly rapid, the crest actually appeared at certain points earlier than at other points further upstream on those same rivers. In the valley from Hamilton north there was actually stored at one time during the 1913 flood a third of the entire rainfall of the storm over the tributary drainage area.

This situation should be kept in mind as having a bearing on the cost and feasibility of channel improvement on the Miami; for in order to prevent overflow in the valley in case of a storm

NOTE. Discussion of this paper is invited, to be received by W. L. Butcher, editor, 14 Beacon St., Boston, before April 10, 1917, for publication in a subsequent number of the JOURNAL.

* Chief Engineer of the Miami Conservancy District, Dayton, Ohio.

like that of 1913, the amount of water to be provided for at any point in the channel must be, not the amount which passed by that point during the crest of the 1913 flood, but the 1913 discharge plus the water which was actually stored in the valley above that point.

The surface soil of almost the entire watershed is glacial till, or clay, sand and gravel, ground up and deposited by the glaciers, but not sifted or sorted, as frequently is the case with glacial drift. As a result, the surface is compact and fairly resistant to the rapid infiltration of water.

The Miami Valley is not to any material degree an alluvial river plain. Before or during the glacial period the valleys of its rivers were scoured out to great depths, and then, with the retreat of the glaciers, the great gorges were filled with *débris*. In making borings at our dam sites we find rock exposed on the valley sides, but on the valley floor, borings nearly two hundred feet deep do not reach the bottom of the glacial *débris*. In some places the ancient paths of the rivers are entirely wiped out and covered with high hills of glacial till, while the present valley follows a new path over the former hilltops.

The Miami River of to-day in its ordinary condition is a puny stream winding its crooked course back and forth across the empty bed of a mighty glacial river. It has dug for itself a little channel, large enough, it is true, to carry the ordinary flow, but wholly insufficient for floods. Table 1 shows the capacity of the unimproved channel outside the cities, compared with the flood flow of 1913. At no point will these streams carry a fifth of the flood flow. Within the cities the channel is larger, but still inadequate.

Many people are of the opinion that when the Lord made the earth he had the comfort and convenience of men particularly in view, and that therefore if a river overflows and causes damage, men must be to blame; they must have obstructed the channel or have occupied the area which rightfully belongs to the river. As a matter of fact, a river is an accidental affair—simply the path water follows when it answers the call of gravity. Long repetition commonly results in the digging of a definite channel, but this is not always the case. The Little River in

TABLE I.

PRESENT CHANNEL CAPACITIES AT VARIOUS LOCATIONS IN THE MIAMI VALLEY
OUTSIDE OF TOWNS AND CITIES, COMPARED WITH THE
1913 FLOOD DISCHARGE.

Stream.	Location.	Channel Capacity. Second- Feet.	Flood Dis- charge. Second- Feet.	Ratio. Per Cent.	Drainage Area. Square Miles.
Mad River.	West of Springfield.	5 000	55 400	9.0	488
Stillwater River..	Above West Milton.....	7 000	86 200	8.1	600
Loramie Creek...	Northwest of Lockington	1 600	25 600	6.3	208
Miami River.....	Below Dayton.....	25 000	252 000	9.9	2 598
Miami River.....	Below Miamisburg.....	35 000	257 000	13.6	2 722
Miami River.....	Below Hamilton.....	25 000	352 000	7.1	3 672
Twin Creek.....	West of Germantown...	3 000	66 000	4.5	272

Missouri, for instance, with a drainage area of about 2 000 sq. miles, absolutely loses its channel and spreads out over great wooded swamps. Most rivers are either underdone or overdone. Take the upper Colorado, for instance. There is a river channel dug so deep that the greatest flood can fill but a small portion of it. The lower Mississippi, on the other hand, never was large enough to carry its flood waters, which cover the lowlands for many miles from its low-water banks. The Miami River channel, like so many others, is large enough to carry only a very small part of the flood flow, and it is left for men to master the unfinished works of nature.

Floods are not a new experience on the Miami. In 1805, some fifteen years after Dayton was settled, one occurred which probably was not exceeded until 1913, although we do not know the comparative discharges of these two floods. That of March, 1913, reached an elevation nearly ten feet higher in Dayton, but it may be that at the earlier date, the valley near the city had been cleared of forest for agriculture, making an excellent flood channel, while few buildings had been erected to obstruct the flow.

Neither do we know which was the greater rainfall. In the wilderness days, vast areas of swamp land held back tempo-

rarily great quantities of water, while the brooks and depressions were choked with fallen trees and underbrush. Moreover, an army of engineers had been at work constructing reservoirs on all the headwaters. In our discussions of the probable increase or decrease of floods we seldom give proper weight to the work of the beaver. In newer countries I have seen the headwaters of streams so thoroughly controlled with beaver dams that perhaps twenty per cent. of entire watersheds of small streams was formed into beaver ponds. As the dams were abandoned they usually broke in the stream beds, leaving true retarding basins above, which held water temporarily during floods. But as to these earliest floods, we can do little more than guess, and in the absence of definite data it would be rash to say that the rainfalls which caused them were less than that of March, 1913.

THE FLOOD OF MARCH 23-27, 1913.

Through the heart of Dayton the 1913 flood was about ten feet higher than any that had occurred for one hundred and twenty-five years. It probably will stand out as one of the great floods of the century. During and immediately after it, Mr. John H. Patterson, president of the National Cash Register Company, did heroic things. At his factory, boats were built at the rate of one every fifteen minutes to rescue those in danger of drowning or burning. Afterward Mr. Patterson financed the city during its period of stress, and in numberless ways furnished those elements of orderliness and purpose necessary to help the city to react in the wonderful way it did. Governor Cox appointed citizens' relief commissions over the state, and they were given powers through quickly enacted state legislation.

The relief commissions appointed flood prevention committees from among their numbers, and it was these flood prevention committees which undertook the problem of preventing another disaster.

It was at this point in the situation, early in May, 1913, that I was first called to Dayton. I found tremendous determi-

nation to stop floods, a feeling that the railroad and city bridges must be to blame and a conviction that dirt must be flying by Fall. There was also a desire to divert as much as possible of the expense to the federal government.

The flood prevention committee was informed, first, that they probably had overestimated the effect of local obstructions upon the flood situation and that the problem probably was not so easily disposed of as they had supposed; second, that no plan of improvement should be adopted without a thorough analysis of the situation, that six months probably would pass before even a provisional solution could be offered, and probably a year before definite plans would be made; third, that federal aid might probably be a will-o'-the-wisp, and that if they wanted flood control they should make up their minds to get it and pay for it, for otherwise they might repeat the history of Pittsburgh and other ill-fated cities which had waited for help.

It is remarkable that in half a day the policy of the flood prevention committee was reformulated and defined, and arrangements made for working out the safety of Dayton and the Miami Valley. The flood prevention committee saw the possible need for coöperation with other cities, and prepared channels of intercourse if they should be needed. I have never met a group of men so quick to see the point, to make accurate appraisals and to define sound policies. The progress made toward flood control in the Miami Valley is due largely to the members of this committee and to the people of the city, who believed in them and backed them up. The committee, seeing that it takes money to get results, in a remarkable campaign of two weeks in the devastated, mud-soaked city raised a fund of \$2 150 000 from 23 000 subscribers, as a beginning toward flood prevention.

One of the happiest results of the flood and of the heroic work following was the breaking up of lifelong prejudices and clans, and the welding of the city into a fellowship of progress. The present city manager government of Dayton, while considered before the flood, is really one of the by-products of this change.

PRELIMINARY INVESTIGATIONS.

The task of working out plans for flood control was assigned to the Morgan Engineering Company of Memphis, Tenn. Field work was begun within two days, in two weeks about fifty engineers were engaged in field and office, largely recruited from flood prevention projects in the southern states, and this force was further enlarged during the course of a month.

The work was begun on the principle that no method of improvement should be adopted which had not been thoroughly tested by comparison with every other method or combination of methods. It therefore became necessary to investigate at one time every method of relief which offered even the least hope, and to collect data bearing on every phase of the problem. Field surveys and office investigations were organized on this basis. We investigated channel improvements, channel cut-offs, diversions into other watersheds, levees and reservoirs. Retarding basins as finally worked out were a development of the investigations into storage reservoirs. Personally, I believed we would find our way out by means of channel improvements, and the investigation into control by means of dams was undertaken from a sense of duty, in order that no possible method might be overlooked.

The very first field work consisted in marking and recording the high-water marks as pointed out by residents along the rivers, and in recording their statements as to the time of the flood crest. The chief fact brought out by the hopelessly conflicting records secured is that the average untrained person is extremely unreliable in observing even the most obvious phenomenon. The high-water profile was later worked out with accuracy by engineers who carefully examined water marks on trees and on the insides of buildings, and the time of the passing of the flood was determined closely at a few points by persons who made definite written records at the time. One of the most important parts of the early field work was the estimation of the actual discharge of the principal streams at the crest of the flood.

While the field work was under way, the field notes were platted, and other investigations were carried on. All available

data were secured as to roads, railroads, artificial drainage and the locations of city improvements. A careful study was made of the rainfall of the 1913 flood, and of previous floods on the Miami. An investigation to locate and study every existing record of important rainstorms in the United States east of the arid regions occupied a force of four men for nearly a year.

Run-off investigations were made at the same time. Rainfall and stream gaging stations were established in all parts of the Miami drainage area. Daily reports are received from all of these, and hourly reports during floods or heavy rains. A thoroughgoing system of flood warnings is maintained. This will be especially useful during construction. Flood stages can be foretold within close limits from the rainfall records, twelve hours or more before their occurrence.

Experimental plots of ground were marked off representing hill land and flat land, both in grass and in cultivation, and the entire run-off from them is caught in subterranean tanks set at the lower corners of the plots. Comparison of this run-off with rainfall collected in rainfall gages at the same location gives valuable data and confirms the data secured on a larger scale from stream gagings and general rainfall records.

One engineer was employed for a year in New York and Washington libraries, translating and extracting European literature in German, French and English dealing with flood control, rainfall, dam construction and hydraulic problems. A most thorough investigation was made of formulas for velocity of water in open channels, both in the literature of the subject and by very carefully conducted experiments.

Measurements of the 1913 flood flow were most interesting. They were made by two entirely separate methods. First, the flood plane was taken as a channel, and formulas for the flow of water in open channels were applied. The surface slope was closely determined by marks on the insides of buildings, and by picking away the outside bark of trees, uncovering the muddy deposit beneath.

For the channel section, Kutter's formula was used, the adopted roughness coefficient being based on my judgment, founded on former measurements, checked by other members of

our force. The roughness factor was fully determined before measurements were calculated, so there would be no unconscious "fudging" of results. The results of these two entirely different methods of measuring the flood flow varied by less than 10 per cent. Similar measurements and checks along the rivers and tributaries have convinced us that our results are within 10 per cent. of accuracy.

TENTATIVE RESULTS.

At the end of six months of work we had arrived at certain conclusions. Among them were:

That a larger flood than that of 1913 must be guarded against. At that time we were inclined to hold that a flood 50 per cent. greater should be our criterion. Later, on further study, this was reduced to 40 per cent.

That channel improvement, if at all feasible, would be very difficult and would have very serious drawbacks.

That all methods except channel improvement, cut-offs locally at Dayton, and retarding basins, were entirely out of the question.

That the Mad River could be diverted into Little Miami, but that such a course would only transfer the difficulty.

That retarding basins apparently were feasible, at no greater cost for saving the valley as a whole than would be the cost of channel improvement for Dayton alone.

That more data were needed before definite conclusions could be reached.

That the problem was one for the valley as a whole and not for each locality working by itself.

That united support from the whole valley was needed.

That there was no law in the state under which the work could be done.

BUILDING A FLOOD CONTROL STATUTE.

The work could not be constructed without a law to work under. The undertaking being in the hands of engineers, the problem of preparing a law was approached from the engineer's point of view. I first made a schedule of all the results that, from an engineer's standpoint, such a law should make possible. Some of these considerations were:

There must be wide freedom of action to carry out any type of improvement and to do any reasonable thing which should be required by plans for flood control.

As the most efficient and effective agency in our country to-day for getting things done is the corporation, it seemed that such improvements should be handled under corporate forms and management.

There must be absence of politics.

A flood-control organization must have governmental powers — as, right of eminent domain, police power, taxing power, etc.

It must fit without friction into existing governmental machinery.

It must make possible all methods of coöperation with corporations, cities, counties, with the home state or other states and with the national government.

It must be fair and not arbitrary.

It should provide control over stream obstructions and over the ownership and use of water.

It must be a general law, suitable to any situation, and so comprehensive that it will not need amendment to fit some other situation that may arise in some other part of the state. (Special laws are not constitutional in Ohio.)

All these needs and many others were covered in the bill that was written. The experience of the Morgan Engineering Company in handling from seventy-five to one hundred water-control projects under fifteen or more state laws, each one with its legal difficulties to overcome, gave a background of experience. American and European water-control laws were carefully examined section by section to see whether some other situation had developed legal provisions which might some time be needed in Ohio. The engineer scorns being bound by precedent, but not so the lawyer or the courts. And so for almost every provision of the bill some good precedent was found in some law of some other state, in order, as Tom Sawyer would say, to do it in the regular way.

The law provides for the organization, upon the petition of landowners, of conservancy districts, these districts to be in effect both political subdivisions and corporations, governed by boards of directors, who have the powers both of the directors of corporations and of governing officers of political subdivisions.

These officers, appointed by a court, so as to be as nearly as possible free from politics, are as unhampered in their work as the directors of a manufacturing company, but have the powers of the governing body of a city or county, so far as flood control is concerned. There is abundant provision for the acts of the board of directors to be reviewed on their merits by the courts.

The engineers furnished the substance and general structure of this law, but it is to the credit of the attorneys of the district that it was so well drafted into form and so carefully guarded that it has been upheld fully by the supreme court of the state. There were many conferences before the things the engineer said must be done were so stated that the constitutional lawyer said they could be done. Then we finally went over the bill in great detail to see that it was as good an example of English and of clearly expressed purpose as it was of law and of engineering.

The bill had a stormy time in the legislature, but was passed, and later sustained intact through another legislative session. Opponents of the law claimed it to be a covert scheme of water-power interests.

It is unnecessary to tell how public opinion was awakened and informed and how effort was made to have the valley as a whole, a narrow strip of country, a hundred miles long and a mile or two wide, consider itself as one community for flood control. Many local officials found themselves deprived of the opportunity to spend money for local flood control, and consequently there was opposition. Yet when an effort was made at a following legislative session to amend the law to its detriment, in three days a petition was signed by 87 000 people in the valley, protesting against any change.

Under this law the Miami Conservancy District was organized in July, 1915, with three prominent business men as directors. Mr. E. A. Deeds, chairman of the Dayton Flood Prevention Committee, became president of this board. More than to any other factors, the progress of the undertaking has been due to his unusual insight, almost unlimited energy, enthusiasm and public spirit. The forces of the district are now housed in a beautiful office building given by Mr. Deeds at a

cost of \$100,000. In February, 1916, a complete plan was submitted to the board of directors. After approval by the board, the plan was reviewed for two months by the Conservancy Court, consisting of a judge from each of the nine counties in the district, and was finally approved, the judges being unani-

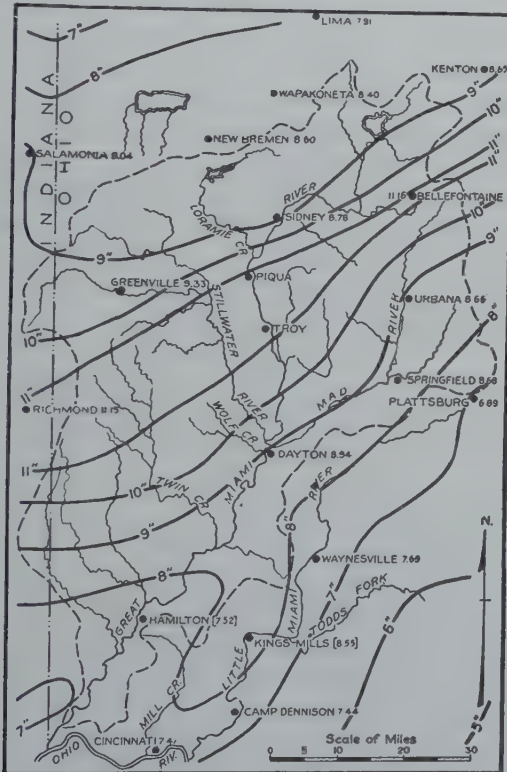


FIG. 1. RAINFALL OF THE STORM OF MARCH 23-27, 1913.

mous on the main point that the retarding basin system, supplemented by channel improvement, is the only feasible method of flood control in the Miami Valley.

PREPARATION OF PLAN.

During all this time the engineering work had steadily progressed. I will briefly review some of the results.

First as to rainfall. A most extensive investigation of flood rainfall is being completed which will be published during the coming year.

It was found that nearly all heavy rainstorms in Ohio came from the southwest. The map (Fig. 1) indicates the rainfall of the storm of March 23-27, 1913, most of which fell in three days. By the way, it appears that in the central United States, a continued intense rain never lasts as long as three days.

The greatest storm so far recorded in this section occurred in October, 1910, and had its center at the southern extremity of Illinois. Nearly all great rainstorms occur during the warm months of the year when the capacity of the ground to hold water is greatest. No winter or early spring rainfall heavier than that of March, 1913, has been recorded east of the Mississippi and north of the Gulf or South Atlantic states.

It was decided to base protective works on an assumption of a flood 40 per cent. greater than that of 1913. This would require a summer rainfall perhaps 60 or 70 per cent. greater than 1913. In view of our limited knowledge of rainfall, it was decided to build dams safe against a flood twice as great as that of 1913.

RELATION OF RAINFALL TO RUN-OFF.

During the 1913 flood, about 87 per cent. of the rainfall ran off and the storage in the river valleys during the crest of the flood was about one third of the total rainfall.

Summer rates of run-off are much lower than winter rates. At any one time the surface soil of the Miami Valley has capacity for temporarily storing about $2\frac{1}{2}$ ins. of rainfall. During the summer this storage capacity is quickly made available by evaporation and transpiration of plants.

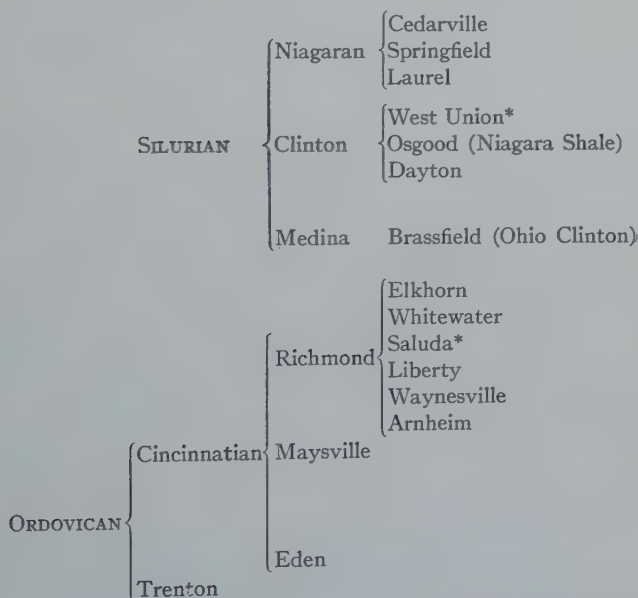
Most painstaking comparisons were made of various velocity formulas. The results of this investigation are being prepared for publication. Kutter's formula, on the whole, stands the tests better than any other, and when used with good judgment and carefully made observations is a dependable engineering aid. A large part of the published observations of stream flow for the use of the formula are so faulty in theory

and in execution as to be nearly valueless. The loss of head at bridge openings and around bends in channels was investigated and will form the subject of a technical report.

In investigations of scour and deposit by running water it was found that velocities of 8 to 10 ft. per second do little damage in the Miami River, but that velocities of 14 to 15 ft. are seriously destructive. In new cuts, exposing soils not worked over by the river, velocities of 6 ft. per second are very erosive.

Comprehensive and thorough tests have been made of the materials of construction. The limestones, sands and gravels of the valley were made into concrete and thoroughly tested. Much of the limestone commonly used for aggregates is slightly oil-bearing and makes poor concrete. The gravel has been thoroughly tested by glacial action and is very satisfactory. All our sand must be washed to develop its full strength.

GEOLOGICAL SCALE, SOUTHWESTERN OHIO.



NOTE. * Indicates formation not found locally. () Indicates name frequently used locally.

FIG. 2.

The geological scale (Fig. 2) covers all the formations met with on our construction. The conduits and spillways on four of the dams are founded on the Cincinnati, consisting of alternating layers of crystalline limestone and of clay or shale. The clay or shale entirely prevents seepage and the development of cavities. The Lockington spillway is founded on Cedarville limestone, which may have cavities. The geological range is from the Ordovician through the Silurian: In the gravel of the drift there is a large amount of glacial-borne granite of Canadian origin.

An elaborate system of borings and test pits has fully developed the foundations for the dams and spillways, and the location of suitable material for earth dams and for concrete construction. Definite mechanical analyses of these materials have been made.

Investigations were made as to the possibility of flood control by means of many small basins instead of a few larger ones, but relief by this method was found to be impracticable.

Check dams to restrain gravel in its movement downstream were investigated, and may be built later in a limited way on tributaries emptying within or near cities.

Channel improvement as a sole method of relief was found impracticable because of enormous first cost, high cost of maintenance in the shifting gravel river beds of the district, the great damages which would result from channel enlargement in cities, and the increase in discharge which would result from eliminating the valley storage such as existed in 1913.

Storage reservoirs such as those recommended at Pittsburgh were rejected, because the same space cannot be made to serve acceptably for both storage and flood control.

THE PLAN.

The plan finally adopted consists of a system of five retarding basins, located on the principal streams, supplemented by limited channel improvement through the cities. The dams which form these retarding basins are so designed as to let pass as much water during flood as can be carried safely through the cities below. There are no gates or other regulating devices.

TABLE 2.
DIMENSIONS OF DAMS AND APPURTENANCES.

	German- town Dam.	Englewood Dam.	Lockington Dam.	Taylorville Dam.	Huffman Dam.
Length of crest of dam, feet.....	1 210	4 660	*6 400	*2 980	*3 340
Thickness of dam at average elevation of valley floor, feet....	650	740	410	390	380
Number of conduits...	2	2	2	4	3
Total sectional area of outlet conduits, square feet.....	182	214	158	1 116	705
Maximum discharge of conduits in 1913 flood, second-feet....	9 340	11 000	8 630	51 300	32 600
Length of spillway crest, feet.....	55	100	72	132	100
Yardage of earth in dams.....	865 000	4 000 000	1 135 000	1 235 000	1 655 000
Yardage of concrete in dams and appurte- nances.....	20 000	38 000	38 000	47 000	40 000

* Including spillway crest.

TABLE 3.

TOTAL RUN-OFF IN INCHES WHICH WILL JUST FILL BASINS TO SPILLWAY
LEVEL IF ENTERING DURING THE RESPECTIVE TIME-INTERVALS SHOWN.

Basin.	0 Days.	1 Day.	2 Days.	3 Days.	4 Days.	5 Days.
Germantown...	7.30	8.45	9.60	10.75	12.13	13.51
Englewood.....	8.99	9.56	10.13	10.70	11.38	12.06
Lockington....	6.68	7.78	8.87	10.00	11.31	12.62
Taylorville*...	4.42	6.12	7.82	9.51	11.52	13.57
Huffman.....	4.66	6.28	7.90	9.52	11.47	13.42

* Taylorville figures are for the area of 778 sq. miles above the Taylorville dam and below the area of 355 sq. miles above the Lockington and Lewistown dams. In making the computation it was assumed in each case that the Lockington basin was filled exactly to the spillway level.

The Germantown dam (Fig. 3) is short and high and causes flooding in a narrow valley. Its spillway and conduit are separated, this being made necessary by the topography. The Huffman dam is of another type, being long and low and causing

flooding to a shallower depth over a broad area. It has a combined conduit and spillway.

The economic balance is reached in the design of a retarding basin dam and its outlets when the water in the basin just reaches the spillway during the most extreme flood. If the water does not reach the spillway under such conditions, the capacity of the basin is not being fully developed. The capacity of the conduits in such case might better be restricted further, so as to give more complete protection below. On the other hand, if water flows

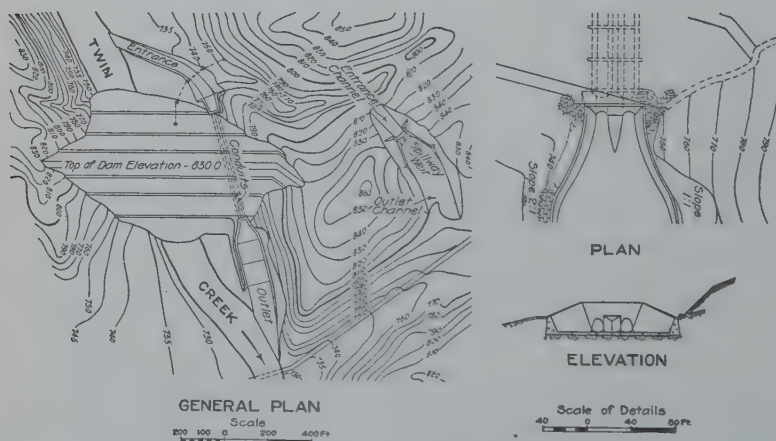


FIG. 3. THE GERMANTOWN DAM.

over the spillway to any appreciable extent, the flow over the spillway is being added to the maximum discharge through the conduits, creating a short secondary crest in the channel below. In such case it would be better to enlarge the conduits, allowing a somewhat larger flow throughout the whole flood, thus keeping enough storage space available to prevent the short secondary crest due to overflow of the spillway. It was first thought that a spillway with an earth crest which would wash out like a movable dam would give an added factor of safety to the dam. But this was abandoned in favor of greater freeboard on the dam, because of its tendency to produce a secondary flood crest.

In general, the dams are designed so that this maximum

efficiency is reached in case of a flood 40 per cent. greater than that of 1913, representing a run-off of ten inches in three days. If a flood 60 per cent. greater should occur, the efficiency of the system would not be quite so great, because of the secondary crest due to overflow over the spillway. This policy of having the maximum degree of control in case of a flood 40 per cent. greater than that of 1913 could not be adhered to in case of the Germantown and Englewood dams, as the necessity of having the dams safe in case of a flood twice as great as 1913 in these cases results in giving these basins their greatest efficiency in a flood more than 40 per cent. greater than 1913.

It has been objected that retarding basins are an innovation. The Rhine at Lake Constance, the Rhone at Lake Geneva and the Po at Laga Maggiore have their flood flows reduced to a very small fraction of the inflows into these lakes. In Germany and Austria, in recent years, extensive systems of retarding basins have been constructed.

In the United States, the Little River Drainage District uses the principle to reduce by a half a flood flow of more than 100 000 second-feet. At Harrisburg, Pa., and at Watervliet, N. Y., the system is in use on small streams. While the particular assemblage of parts adopted in the Miami Valley has no precedents on a similar scale, yet every element has been thoroughly tested on as large scale in other constructions.

SOME PROBLEMS ENCOUNTERED.

Many interesting problems have been worked out in the preparation of the plans. One of the most interesting is the method of destroying the velocity issuing from the conduits under a high head. In considering this problem, an experimental plant was constructed, duplicating the Germantown conduit outlets on a one-sixteenth scale, and using velocities one fourth of what would exist in the actual construction. Baffles somewhat like those at Gatun* were tried and found to be very ineffective. A part of the water weaves its way between the baffles like a snake crawling between stalks of shrubbery. After a year's experi-

* JOURNAL B. S. C. E., Vol. 3, p. 135.



FIG. 4. TYPE OF OUTLET FROM CONDUITS.

ments the type of outlet shown in Fig. 4 was chosen. The water emerging from the conduit falls down the inclined plane and meets a weir over which it must pass. Back water from below tends to pile on top of it and a standing wave is created, destroying the velocity energy by internal friction. A view of the standing wave in action is shown in Fig. 5.



FIG. 5. STANDING WAVE AT FOOT OF OUTLET.

The causes of earth dam failures were thoroughly studied in detail. While failures of poorly designed dams are numerous, in nearly every case the cause was found to be one of the following: Overtopping, sliding of slopes, leaking through the dam, seepage under the dam, wave action, back lash from the outlet below the dam, seepage along conduits, failure or settlement of outlet works, burrowing animals, malicious acts or neglect.

We have taken up these causes of failure one by one, and have so designed our works that we believe we are absolutely safe against failure from any one of these causes or from any other short of a great earthquake or some other cataclysm.

The responsibility for the safety of the work has rested heavily upon the engineers. That is the reason why the factors of safety chosen are so enormously large, why the structures have been made massive to the point of being open to serious criticism, and why every least detail has been painstakingly worked out, as though the safety of the entire system depended upon that detail.

And all through the design our patron saint has been — not the Greek architect who could design a beautiful structure, graceful and slender, but rather Pharaoh, who built his pyramids on so broad a base that no matter what mistakes of judgment might be made, or how faulty work might be done in the building, they would yet stand through the thousands of years.

BOSTON SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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DISCUSSION OF "THE GROINED ARCH AS A MEANS OF CONCRETE FLOOR CONSTRUCTION."

BY MESSRS. CHARLES R. GOW, FRANK L. FULLER, E. F. ROCKWOOD,
ALLEN HAZEN, JOHN R. NICHOLS AND H. WHITTEMORE BROWN.

MR. GOW.*—I am disposed to feel that there is a great deal of merit in the suggestion which the author has brought forth in this paper. From my experience with this form of construction, I have been very much impressed with the strength and the possibilities in the use of the groined arch. For example, in the construction of the Springfield filters to which the author referred, we had a crown thickness of 6 ins. and a 13-ft. span, with a depression over the columns, as is customary in such work, for economy. We found it was possible to support our operating derricks on the roof with hoisting engines and other apparatus. At first we were rather timid, and always placed the loads over columns, but as time went on and nothing happened we became bolder and subsequently would place the foot block of the derrick or land the engine at any point, on the crown or elsewhere, and at times there would be a concentrated load of perhaps eight or ten tons under a derrick support, and during the entire construction there was no sign of weakness. We often had occasion to drive over the roof with loaded teams, and while the specifications called for a protecting covering of a few inches of earth, we did not always get it, and it was a common occurrence to have a loaded team weighing three to four tons

* The Charles R. Gow Co., Contractors, Boston.

roll over the crown of these arches and perhaps over a good-sized cobblestone and bang down again, and even under that rough usage there was never a single instance of failure or cracking. It became apparent, therefore, that this method of construction, if adequately supported against lateral spreading, was exceptionally strong. We were, of course, always apprehensive of the danger of a failure due to spreading by reason of the premature removing of the supports at either end, or a failure sufficiently to back up the abutments, and, on at least one occasion, we had a near failure due to stripping forms too close to the working end. It was our practice to keep at least three bays of forms always in place under their posting, to insure adequate resistance at that point.

About a year or so ago, we had a similar piece of construction at Lowell, Mass., and in the case of part of the work had a special design of roof, that is, the spans were not standard and it required a separate set of forms. This work was not extensive enough in volume to warrant a large outlay for forms, and we found progress very slow because we had to keep so large a percentage of the forms in place as a support for the thrust action. At the suggestion of Mr. Frank A. Barbour, a member of our Society, who was consulting engineer for the City, we reinforced the squares of the roof about each column with four $\frac{1}{2}$ -in. round bars, and when I refer to squares I mean the square formed by the four sections of forms which surrounded each column, — in other words, a section of column and roof which resembles an umbrella turned partly inside out. These squares were used as concreting units, and it was our practice to place the four $\frac{1}{2}$ -in. round steel bars on the perimeter of that square. We then found it was possible to drop the forms entirely after the concrete had set. We didn't do this in the beginning, but as we grew bolder we reached the point where we would take away every bit of form and set it ahead, leaving the last half arch cantilevering out from its column. During the entire work there was no sign of stress. The self-supporting quality in the arches apparently resulted from a double action of cantilever and circumferential hooping, producing a form of ring tension.

With regard to the cost of the forms that the author has noted, I find it was true that in the case of the Springfield work the arch forms cost four and one-half cents per square foot. In the Lowell work, which was not quite so extensive, they cost six cents. The bulk of this increase was undoubtedly due to the time that had elapsed, — I think some seven years, — and I have no doubt if the same work was to be done to-day it would cost very much in excess of the higher figure, carpenters' wages having increased materially as well as that of unskilled labor, with some increase also in material costs. I think, however, that the relative cost of various types of forms would presumably remain about the same.

It is true that groined arch form work is very much cheaper than is usually supposed. It looks complicated on paper and also somewhat complicated after it is constructed, but nevertheless if the work is intelligently executed it is a very economical and simple type of form work. One of the reasons for this economy is the facility with which the sections of groined arch forms can be erected, removed and used over a large number of times, which, generally speaking, is not true of beam and slab or mushroom type of construction, the form work of which ordinarily is largely or entirely dismantled before it is put together again. The groined arch forms can be erected and removed intact, with very little repair, and there isn't the same waste of lumber. The specially curved form sections used in groined arch construction are practically worthless for any other purpose and are therefore not diverted to various uses, as is the case with other types of forms after they have been dismantled.

Until to-night, when I heard the speaker say that he had designed a span of 20 ft. (I think he said 20 ft. x 20 ft.) with a total rise of 25 in., I had wondered if this type of construction wouldn't be prohibitive on the large spans encountered in ordinary building construction, but, if it is true that it can be accomplished in a vertical distance of 25 ins., these spans may not be as prohibitive as it seems at first glance.

On the whole, I am disposed to believe that the suggestion which the paper brings to us has much practical value, even

though its use be limited to special rather than general application.

MR. FULLER.* — I have covered two open reservoirs with this form of roof and they have been entirely satisfactory. On one at Natick, the only trouble we had was in the sections where the outer part of the arches rested upon the embankment. We had a core wall which we knew nothing about, it having been built a good many years before, and when it became necessary to cover the reservoir, we placed the piers so that there should be a row about midway of the embankment, and the outer edge of course had to rest on the top of the bank, and we excavated far enough to get to this core wall. That didn't prove to be entirely satisfactory. There must have been some slight settlement of the wall, because we had, in places, cracks between the edge of the embankment and the first row of piers, and parallel with them. That was afterwards remedied by building up some supports on the under side from the paving, which was a very good piece of work, and, so far as I know, there has never been any trouble since, but in the entire roof within the four lines of piers I never saw any cracks or any settlement or any trouble whatsoever. In the case of a heavy rain, a slight leakage occurred, where different sections of concrete joined, but that we didn't consider as anything serious and made no special effort to stop it. As in Mr. Gow's instance, the roof was subjected to very hard usage. It wasn't completed till rather late in the season, and a few inches of earth was put on to protect it from the frost, and after a heavy rain we had some very cold weather before the material was spread very much. Of course it was dumped by carts and spread a little. These carts went right over the concrete, but some of it had not been laid very long and was not entirely dry. The material was left in ridges more uneven than the furrows of a field. It was thought desirable to finish the covering that season and a new covering had to be put on over the frozen material. By building a runway and using block and falls to help up the teams, very heavy loads of earth were carried on to the roof and of course had to be driven over it and taken to different parts. It was extremely rough,

* Civil Engineer, Boston.

and those teams went bumping over that frozen material in a manner which made one almost feel that something would give way, but it never did and the roof has stood perfectly well. At Wellesley we covered a similar but smaller reservoir in the same way. There I arranged the piers so that they should come nearer to the edge of the bank, and we had less width and less weight of concrete between the outer row of piers and the top of the bank, and so far as I know there have never been any cracks in that reservoir roof. Two years ago we built a covered reservoir for the town of Webster, using the flat slab construction, which also was very satisfactory and gave no trouble whatsoever. I have never figured out the comparative cost of the two methods, but I do not believe they would vary much. In either of these cases it would be difficult to know just what the cost per square foot of centering was, because the cost was included in the price per cubic yard of concrete of which the roof is composed, but, as has been said, the groined arch centers are not very difficult to make and they can be used over and over again and can be removed after having been sufficiently long in place with little damage. I think that although there would be perhaps more carpenter work in a given surface, or a given area, the fact that these centers can be used over and over again, and the fact that probably in flat slab construction it would be better to center the whole reservoir (I don't know how it would be in a building), the cost of the two methods of centering might not be very different.

The old Natick open reservoir, with sloping embankments, outside and in, was covered in 1902. There are thirteen rows of piers, with thirteen piers in each row, or 169 in all. Piers are 15 ft. 2 ins. on centers and are 20 ins. square above the base. The area of the roof is 44 700 sq. ft. and the capacity of the reservoir, 4 280 000 gals. The elliptical arches used in the design of the roof have a span of 13 ft. 6 ins. and a rise of 2 ft. 9 ins. The thickness of the concrete roof at the crown is 6 ins. and over piers, 3 ft. 3 ins.

Twenty-two hundred barrels of Atlas cement, 200 of Brooks-Shoobridge, and 200 of Alsen American Portland cement, or 2 600 barrels in all, were used. The concrete was composed of

one volume of cement, two and one half of sand and four and one half of screened gravel, mixed very wet.

Concrete for the roof was put on in sections 30 ft. 4 ins. wide, and of a length equal to the width of the reservoir, the joint being along the crown of the arch. Through this joint there was some leakage in times of heavy rain, but this caused no particular harm.

The minimum time allowed for removing pier forms was about four days. Generally they were allowed to stand longer. The minimum time allowed for removal of roof centers was ten days. In some cases they remained two weeks or longer, or until needed.

The work was done by contract by Mr. F. A. Snow, of Boston.

The cost of concrete on the bottom was \$8.50 per cubic yard.

"	"	"	"	in piers	"	9.50	"	"	"
"	"	"	"	" roof	"	12.50	"	"	"

Total amount expended in reservoir improvements, \$30 116.75.

MR. ROCKWOOD* (*by letter*). — The writer has read with a good deal of interest the paper on the groined arch as a means of concrete floor construction and it would seem to him that one very serious drawback to this type of construction would be the difficulty in attaching inserts to the forms or hanging shafting, piping, etc., from the under side of the slab.

In regard to the cost, the writer would question whether the costs given for this type of construction as compared with beam and slab or mushroom construction are both based on the same conditions, whether the unit prices for materials and labor were the same, whether the story heights were similar, whether the forms were used over the same number of times, and whether both costs included initial cost as well as erecting, stripping, etc.

MR. HAZEN† (*by letter*). — The writer is pleased to see a discussion of the use of the groined arch in ordinary floor construction. Long experience with it in covers for filters and pure-

* Chief Engineer, New England Concrete Construction Co., Boston.

† Consulting Engineer, New York.

water reservoirs leads him to believe that there is much merit in the suggestion.

For one thing, it puts all the concrete in compression and develops that admirable quality of concrete in the most advantageous manner. Reinforcing is unnecessary, except that a moderate amount of reinforcing may be convenient during construction in permitting forms to be more quickly moved forward and to serve afterwards as an additional precaution against failure under extreme unequal loading, or in case of slight movement by settlement of foundations or otherwise.

As the author points out, the cost of the forms is probably no greater; it even seems to be less in some cases than the cost of forms for slab and beam construction. The volume of concrete, also, with a good groined arch design is surprisingly small when compared with that required for slab and beam construction, and the absence of reinforcing is an unbalanced item of cost in favor of the groined construction. The thrust of the concrete arch must be adequately provided for, and when this is done the groined arch is extraordinarily strong. This is demonstrated by tests such as those made by the author and also by wide practical experience.

The difficulty of computing the stresses in a groined arch is great, and this difficulty has no doubt tended to retard its adoption, because designers have been reluctant to adopt a type of construction, the stresses in which they could not compute with a reasonable degree of certainty, even though practical experience with it was favorable.

The writer has given this question of stresses much consideration. Probably the safest general method is along the lines referred to by the author and attributed to Mr. Horace H. Chase, but the writer has carried it a little further. The whole arch, instead of being divided into quarters, may be divided into very narrow radial segments, each joining and opposing an equal and opposite segment springing from the next pier. Computation made in this way neglects the obvious fact that some of the segments are stronger than others, the strongest ones being on the straight lines between the piers and the weakest ones being the diagonal ones, and that the general stiffness of the

structure will partially transfer some of the surplus strength of the straight arches to the diagonal ones. Neglecting this consideration, the unit stresses in the diagonals will be just twice as great as they are in the shortest arches between two piers.

This suggests a practical short-cut method of estimating stresses which the writer has followed and which has seemed to be safe in his work.

First, the radius of a circle is found, having approximately the same curvature as the arch at the top on the short line between two piers. The elliptical form of the interior has been always used in our work. There is no theoretical reason for using the elliptical form, but in the analysis of stresses that we have made by various methods, and the following endeavor to draw an arch to best fit them, the shape found has worked out to be so near an ellipse that there was no reason for departing from it. The use of the ellipse makes laying out easy and calculating volumes easy, but that is not a sufficient reason if the analysis indicated any other form.

With an elliptical interior the radius of a circle of equal curvature at the top is the square of the half span divided by the rise.

The exterior of the arch in our work, ordinarily parabolic, has a somewhat greater radius of curvature at the top, ordinarily from 10 per cent. to 20 per cent. greater, and in a general way the radius of a circle of equal curvature lying in the middle would be from 5 per cent. to 10 per cent. greater than that of the ellipse forming the lower surface.

The unit pressure at the top of a cylindrical arch is the weight of a unit of unit area multiplied by the radius; for instance, if we have 6 ins. of concrete weighing 75 lbs. and 2 ft. of earth weighing 200 lbs., and with a small allowance for other occasional loads, making a total of 300 lbs. per sq. ft., and if the average radius of curvature is 20 ft., the thrust at the top (assuming it to be a cylindrical arch) is 6 000 lbs. per linear foot, or 12 000 lbs. per sq. ft. on the half square foot of concrete exposed in one linear foot of arch, equal to 83 lbs. per sq. in. on the whole area. On the diagonal section, this thrust will be doubled, and will amount to 166 lbs. per sq. in.

It is then further assumed that the whole thrust is not equally distributed on the whole area of the concrete, but that it is concentrated in some measure at one side or the other, and because of this it is probably somewhere double the average, or 333 lbs. per sq. in.

Designs have been made so that, with all reasonable assumptions of temporary and extra loading (not included in the above example), the unit pressures first computed have not exceeded 150 lbs. per sq. in., giving a final ultimate thrust of 600 lbs. per sq. in. at some places. With a radius of curvature at the top of 20 ft., this corresponds to a loading of 540 lbs. per sq. ft.; in other words, starting with the assumptions made above, the design would still be safe with 30 ins. additional earth fill over the whole top, or it would be safe with a smaller unequal loading.

This method of calculation, while empirical, seems to the writer to give reasonable results as tested by all the calculations that he has been able to make and by experience with groined arches designed in this way.

It may be pointed out that making the top of the arches flat, as will be the case in a floor, gives any section which is not heavily loaded a power to act as a strut between adjoining sections that are loaded, in a way that does not exist in ordinary reservoir or filter construction, and such a floor, especially if stiffened with a moderate amount of reinforcing, should be very much better adapted to carry unbalanced loads.

Expansion due to temperature changes may require consideration. In filter and reservoir roofs the barrel arches always used at the sides, and which therefore completely surround the groined arches, permit a slight temperature movement to be taken up. With a flat top carried to the limits, this outlet for movement would not exist, and in large floor areas some other provision might be found necessary.

MR. JOHN R. NICHOLS.* — Mr. Brown's adaptation of the groined arch to concrete floor and roof construction in buildings is of interest to engineers at least as an example of the greatly varied possibilities of reinforced concrete. To what extent this type of construction will be used in buildings, the future

* With Monks & Johnson, Boston.

only will disclose. It may, some day, be used in storage warehouses and some types of mills as extensively as it is now used for the roofs of reservoirs and filters. But that will be only after the pioneers of construction with courage or recklessness (only the results can determine which) shall have demonstrated its fitness for such use. For the ordinary loft building or factory, the groined arch floor, whatever may be its structural sufficiency, has a defect that is probably fatal. The groined ceiling is not a convenient surface for attaching motors and shafting, and for this reason manufacturers in general will probably not look upon it with favor.

The writer has in mind, however, one case where the groined arch floor may prove exceedingly useful. This is the case of a building, the foundations of which penetrate through a layer of soft soil to firmer soil below, and the ground or basement floor of which rests upon the soft material. The surface of the ground could then be graded by the aid of templates to the shape of the groined arch soffit, and concrete poured over it, producing a constructed floor supported on firm foundations at a cost only slightly exceeding that of the ordinary concrete paving laid on the ground, which, in this case, would be subject to settlement. At least, it seems likely that the cost of such a groined arch floor would be considerably less than the customary reinforced floor which would otherwise be used to avoid settlement.

In the design of groined arch floors as described by Mr. Brown, it is of course necessary not to lose sight of the bending in the columns occasioned by raising the tie rods to the level of the crown. Provision must be made in the interior columns to take care of an unbalanced live load and in the wall column for the live and dead load.

If we have square bays, columns l ft. apart on centers, and the load in question (live for interior columns and live and dead for exterior columns) is represented by w lbs. per sq. ft., then the bending to be provided for in any given column at its junction with the floor is approximately $\frac{wl^3}{8}$.

This bending moment is divided between the section of the column above and that below the floor in a proportion which

is determined, among other factors, by the relative stiffness of the two column sections and their lengths. The amount of this bending and its distribution in the upper and lower columns are also affected by the distribution of live load not only on the floor under consideration but on other floors of the building as well, and in so complicated a fashion that it is impractical to determine the bending in any given length of column with any attempt at precision. The approximate value for the total bending given above ignores continuity and loading of portions of the structure other than that immediately under consideration, but it probably takes care of the worst cases of loading that are likely to occur.

It is by no means impossible or even impracticable to make provision in columns for such bending. We are constantly doing it in columns supporting flat slab construction, though the moments there are somewhat less in magnitude. But the cost of such provision and the floor space occupied by enlarged columns must offset to some extent the advantages of the groined arch construction for the upper floors of buildings.

The value given above for the bending in columns is also useful in determining the stress in the tie rods of the arch itself, with some reduction in l (the span), to take care of the thickness of the columns. In this case, however, w is always the total live and dead load. If l_1 is the span of the arch and a the vertical distance between the tie-rod (horizontal through the crown) and the center of compression in the arch at the springing, then the total tension in the tie rod (or rods) in one direction pertaining to one column or bay is $\frac{wl_1^2}{8a}$.

In conclusion, the groined arch has now a considerable field of usefulness. With the raised tie-rod introduced by Mr. Brown there is hope that an extension of this field into some phases of building construction may be advantageous. But the process will be slow while the conservative (or timid) wait for the reckless (or courageous) to show them the way.

MR. BROWN.* — I believe Mr. Gow's experiences serve to reinforce my contention that the groined arch is very strong,

*Author's closure.

even under concentrated loads, and without steel reinforcement; although it is true that he used a very much greater crown thickness than I am proposing, he finds that concentrated derrick loads of eight or ten tons cause no distress. These are probably much more severe conditions than we would expect to encounter in building work.

The great advantage gained by the use of small amounts of reinforcing steel in the Lowell filter is also of pertinent interest; in the method which he used, there would seem to be a combination of hoop tension and cantilever action which is somewhat different from that which is the case with reinforcing bands around the sides of the bay, as I have suggested.

The design referred to is as follows:

Using the same ratio of rise to span on a 20 ft. x 20 ft. bay as was used in the test described, the constant multiplier would be $\frac{20}{8} = 2.5$.

That is, it would require 2.5 times the area of steel, the rise and the crown thickness in a 20 x 20 bay, as was required in the 8 x 8 test bay in order to give the same strength in pounds per square foot on the floor.

The total weight of steel in the arch would then be:

As there were two $\frac{3}{4}$ -in. round bars used in test floor, there would be required $2 \times 2.5 = 5 \frac{3}{4}$ bars in 20 x 20 floor; length around 20 x 20 bay = 80 ft.; $\frac{3}{4}$ round bar weighs 1.50 lbs. per ft.; total weight = $5 \times 80 \times 1.50 = 600$ lbs., or 1.5 lbs. per sq. ft. of floor.

VOLUME OF CONCRETE.

Average thickness test arch	= 4 ins.
Average thickness 20 x 20 arch, 2.5 x 4	= 10 ins.
Volume = 20 x 20 x 10/12	= 333 cu. ft.

DIMENSIONS, 20 X 20 ARCH.

Rise, 2.5 x 10	= 25 ins.
Crown thickness, 2.5 x 2	= 5 ins.
Total depth at columns	= 30 ins.

The first crack on the test arch occurred at a load of 750 lbs. per sq. ft.; using a safety factor of 3 on this load, the safe load would be 250 lbs. per sq. ft.

Comparing this design with that of a flat slab floor of the same dimensions and for the same loads, i. e., 20 ft. x 20 ft., 250 lbs. per sq. ft. live load:

GENERAL DIMENSIONS.

Slab thickness, 9 ins.

Drop panel, 8 ft. square, 4 ins. thick.

Column cap, 4 ft. 6 ins. diameter at top, 45 degree slope.

VOLUME OF CONCRETE.

Slab, 400 x 9/12	= 300 cu. ft.
Drop, 64 x 4/12	= 21 cu. ft.
Cap (excess outside column)	= 11 cu. ft.
Total	= 332 cu. ft.

WEIGHT OF STEEL.

Fourteen, $\frac{5}{8}$ in. square, 30 ft. 6 ins. long	= 567 lbs.
Fourteen, $\frac{5}{8}$ in. square, 18 ft. 0 ins. long	= 335 lbs.
Ten, $\frac{1}{2}$ in. square, 30 ft. 6 ins. long	= 259 lbs.
Ten, $\frac{1}{2}$ in. square, 10 ft. 0 ins. long	= 153 lbs.
Six, $\frac{5}{8}$ in. rounds, 20 ft. 0 ins. long	= 62 lbs.
Total	1 376 lbs.

which is about 3.5 lbs. per sq. ft. of floor.*

Mr. Fuller's discussion also serves to illustrate the great strength and usefulness of the groined arch.

I admit Mr. Rockwood's contention that the groined ceiling would be open to certain objections from the point of the adaptability of hanging shafting; until some one invents a new way of hanging shafting, we shall be obliged to confine the groined arch to buildings where this difficulty does not obtain.

As to the criticism of the cost data, a little more careful observation of my statements should suffice, i. e., "to be sure, some allowances ought to be made for the increase in cost, due to the height of the buildings; but this [the cost data given in paper] should dispel any fear that the introduction of curved forms materially increases the cost of the form work." I think this statement has been borne out by the statements of Mr. Gow

* I am indebted to the Aberthaw Construction Co. for the slab design given above.

and Mr. Fuller. It was not the object of the paper to give any such careful analysis of costs as contemplated by Mr. Rockwood.

I was very much pleased with the discussion presented by Mr. Hazen, as it not only bears out a great many of my ideas, but also gives a short-cut method of design. Although a proof of his method of design is not given, I believe that it shows that empirical methods of design for arches are not impossible or impracticable. My small experience with flat slab design leads me to believe that results may be obtained with empirical arch designs which will check as closely with the actual conditions as do the results of our variety of flat slab designs.

The consideration of temperature stresses is indeed pertinent. I hope that it may receive attention in the near future.

Mr. Nichols brings out the same point about the difficulty of attaching shafting, etc., to the groined arch ceiling that Mr. Rockwood mentions, and I shall have to give him the same reply.

The magnitude of the bending moment in the columns emphasized by Mr. Nichols must be considered, but need be no cause for rejection, inasmuch as we are not obliged to keep the rods at the top of the arch. If the bending becomes excessive in the exterior columns (which are the only ones, under ordinary conditions, that would require consideration), the rods extending from the first interior column to the exterior column could be inclined downward enough so that they would counteract the thrust at the exterior at about the position of the resultant thrust, thus reducing the bending to zero. This would possibly require the use of a little extra concrete for fireproofing the rods where they might cut through the concrete of the arch. So long as the rods are straight between columns, even though they are not horizontal, the action remains substantially the same as in the arch which I tested.

The method suggested by Mr. Nichols for computing the stress in the tie-rods contains nothing of a very novel character, inasmuch as the position of center of compression at the springing and at the crown (which Mr. Nichols assumes when he considers the center of the tie-rods to pass through the center of compression at the crown) makes the structure statically de-

terminate. It is the finding of these two positions which involves us in the theory of the elastic arch; when that is done, our problem is practically solved.

The disadvantages of the groined ceiling having been pretty well brought out, let us now consider some of its advantages.

We should not overlook the tendency toward individual drives for machinery, made possible by the introduction of electric power, which, obviating shafting, does away with the greatest objection to the groined ceiling.

Also consider the advantages to be gained in lighting and ventilation by the groined ceiling; a light placed at the center has a natural reflector in the curved surfaces of the ceiling, the absence of beams and girders gives excellent opportunities for ventilation.

The great opportunity for a pleasing architectural treatment to be gained by the proper combination of columns, capitals and the groined ceiling should also be considered, as it is a combination which, through its form and distribution of mass alone, will give a most pleasing architectural effect.

In conclusion, I wish to point the fact that those of my commenters who have had considerable experience with groined arches seem favorably disposed toward my suggestions for their application to floors, while those whose experience with them is of a somewhat limited nature would have to be placed in Mr. Nichols' category of the conservative or timid.

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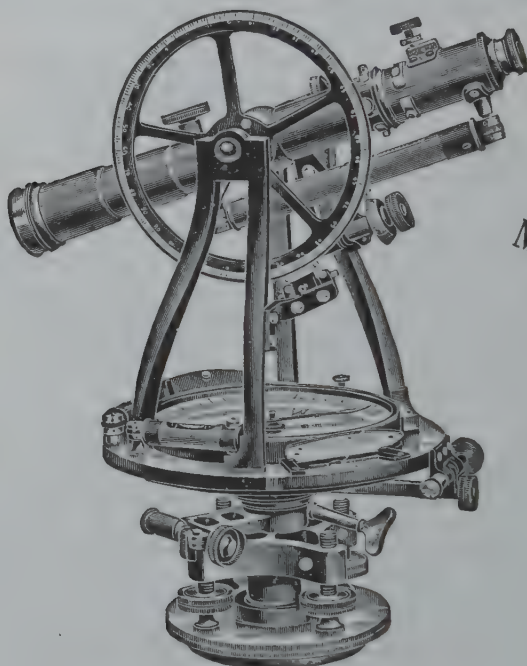
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